6.4.3.1—Bolts

Revise the 3rd Paragraph as follows:

AASHTO M 253 (AASHTO A490) bolts and ASTM A354 Grade BD, bolts, studs, and other externally threaded fasteners, ASTM F1554 Grade 105 (with F_u = 150 ksi) anchor bolts and ASTM A722 bars shall not be galvanized. No cleaning process shall be used that will introduce hydrogen into steel.

C6.4.3.1

Add a 2nd Paragraph as follows:

Galvanization of AASHTO M 253 (ASTM A490) and ASTM A354 Grade BD fasteners, ASTM F1554 Grade 105 (with F_u = 150 ksi) anchor bolts and ASTM A722 bars is not permitted due to hydrogen embrittlement. These fasteners should be carefully used with applicable protective coatings conforming to AASHTO M 253 (ASTM A490) and ASTM A354, ASTM F1554 and ASTM A722 Specifications.

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6.6—FATIGUE AND FRACTURE CONSIDERATIONS

6.6.1.2.3— Detail Categories

Revise Table 6.6.1.2.3-2 as follows:

Table 6.6.1.2.3-2—N_{TH} and 75-yr ($ADTT)_{SL}$ Equivalent to Infinite Life

Detail Category	N_{TH} (Number of Cycles Equivalent to Infinite Life)	75-yrs ($ADTT)_{SL}$ Equivalent to Infinite Life (trucks per Day)
A	1,809,000	~~530~~ 65
B	2,930,000	~~860~~ 110
B′	3,530,000	~~1035~~ 130
C	4,400,000	~~1290~~ 160
C′	2,546,000	~~745~~ 90
D	6,413,000	~~1875~~ 230
E	12,071,000	~~3530~~ 440
E′	22,189,000	~~6485~~ 815

C6.6.1.2.3

Revise the first sentence of the 8th Paragraph as follows:

The values in the second and the third columns of Table 6.6.1.2.3-2 were computed as follows:

$$75\_Year(ADTT)_{SL} = \frac{A}{\left[\frac{(\Delta F)_{TH}}{2}\right]^3 (365)(75)(n)}$$

$$75\_Year(ADTT)_{SL} = \frac{N_{TH}}{(365)(75)(n)} \qquad \text{(C6.6.1.2.3-1)}$$

$$N_{TH} = \frac{A}{\left[(\Delta F)_{TH}\right]^3} \qquad \text{(C6.6.1.2.3-2)}$$

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6.6.1.2.5 —Fatigue Resistance

C6.6.1.2.5

Revise Table 6.6.1.2.5-2 as follows:

Add a new last Paragraph as follows:

Table 6.6.1.2.5-2—Cycles per Truck Passage, *n*

Longitudinal Members		Span Length	
		> 40.0 ft.	≤ 40.0 ft.
Simple Span Girders		1.0	2.0
Continuous Girders	Near Interior Support	1.5 (Fatigue I) 1.2 (Fatigue II)	2.0
	Elsewhere	1.0	2.0
Cantilever Girders		5.0	
Orthotropic Deck Plate Connections Subjected to Wheel Load Cycling		5.0	
Trusses		1.0	
Transverse Members		Spacing	
		> 20.0 ft.	≤ 20.0 ft.
		1.0	2.0

Cycles per design fatigue Permit Truck (Fatigue II limit state) passage are evaluated by the rainflow method. The numbers of cycles induced by the fatigue Permit Truck passage are somewhat similar to the cycles induced by the HL-93 fatigue truck used for Fatigue I Limit State, except in the case of near interior supports of bridges that spans greater than 40 feet.

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6.10.7.1.2—Nominal Flexural Resistance

Revise Eq. 6.10.7.1.2-2 as follows:

$$\cancel{M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right)}$$

$$M_n = \left[1 - \left(1 - \frac{M_y}{M_p}\right)\left(\frac{\frac{D_p}{D_t} - 0.1}{0.32}\right)\right]M_p \qquad (6.10.7.1.2\text{-}2)$$

C6.10.7.1.2

Revise the 2nd Paragraph as follows:

Eq. 10.7.1.2-2 <u>defines the inelastic moment resistance as a straight line between the ductility limits $D_p/D_t = 0.1$ and 0.42. It</u> gives approximately the same results as the comparable equation in previous Specifications, but is a simpler form that depends only on the plastic moment resistance M_p<u>, the yield moment resistance M_y,</u> and ~~on~~ the ratio D_p/D_t~~.~~ ~~as also suggested in Yakel and Azizinamini (2005). Both equations implement the above philosophy justified by Wittry (1993). Eq. 10.7.1.2-2 is somewhat more restrictive than the equation in previous Specifications for sections with small values of M_p/M_y, such as sections with hybrid webs, a relatively small deck area and a high strength tension flange. It is somewhat less restrictive for sections with large values of M_p/M_y. Wittry (1993) considered various experimental test results and performed a large number of parametric cross section analyses. The smallest experimental or theoretical resistance of all the cross sections considered in this research and in other subsequent studies is $0.96M_p$. Eq. 6.10.7.1.2.2 is based on the target additional margin of safety of 1.28 specified by Wittry at the maximum allowed value of D_p combined with an assumed theoretical resistance of $0.96M_p$ at this limit. At the maximum allowed value of D_p specified by Eq. 6.10.7.3-1, the resulting nominal design flexural resistance is $0.78M_p$.~~

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6.10.10.4.1—General

Replace Eq. (6.10.10.4.2-8) as follows:

$$\cancel{P_{2n} = 0.45 f'_c b_s t}$$

$$\underline{P_{2n} = F_{yrs} A_{rs}} \qquad\qquad (6.10.10.4.2\text{-}8)$$

where:

$\underline{A_{rs}}$ = total area of the longitudinal reinforcement within the effective concrete deck width (in.^{2})

$\underline{F_{yrs}}$ = specified minimum yield strength of longitudinal reinforcement within the effective concrete deck width (ksi)

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6.10.11.1—Transverse Stiffeners

6.10.11.1.1—General

Revise the 2nd and 4th Paragraphs as follows:

Stiffeners ~~in straight girders~~ not used as connection plates shall be <u>welded to</u> ~~tight fit at~~ the compression flange <u>and fitted tightly to the tension flange.</u>~~, but need not be in bearing with the tension flange.~~ Single-sided stiffeners on horizontally curved girders should be attached to both flanges. ~~When pairs of transverse stiffeners are used on horizontally curved girders, they shall be fitted tightly to both flanges.~~

The distance between the end of the web-to-stiffener weld and the near edge of the adjacent web-to-flange or longitudinal stiffener-to-web weld shall not be less than $4t_w$, <u>nor more than</u> ~~but shall not exceed the lesser of~~ $6t_w$. <u>In no case shall the distance exceed</u> ~~and~~ 4.0 in.

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6.10.11.2.1—General

Revise the 4th Paragraph as follows:

Each stiffener shall be ~~either milled~~ <u>attached</u> to ~~bear against~~ the flange through which it receives its load ~~or attached to that flange by a full penetration groove weld.~~ <u>by one of the following:</u>

- <u>Milled or ground to bear plus fillet weld both sides,</u>
- <u>Full penetration groove weld.</u>

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6.13.1—General

Revise the 1st Paragraph as follows:

Except as specified otherwise, connections and splices for primary members shall be designed at the strength limit state for not less than ~~the larger of~~:

- ~~The average of the flexural moment induced stress, shear, or axial force due to the factored loadings at the point of splice or connection and the factored flexural, shear, or axial resistance of the member or element at the same point, or~~

- 100 ~~75~~ percent of the factored ~~flexural, shear, or axial~~ resistance<u>s</u> of the member or element.

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6.13.6.1.4b—Web Splices

Revise the 2nd Paragraph as follows:

As a minimum, at the strength limit state, the design shear, V_{uw}, shall be taken as the smaller factored shear resistance of the girder webs at the point of splice, $\phi_v V_n$ ~~follows~~:

- ~~If $V_u < 0.5\phi_v V_n$, then:~~

$$~~V_{uw} = 1.5 V_u~~ \qquad (6.13.6.1.4b\text{-}1)$$

- ~~Otherwise:~~

$$~~V_{uw} = \frac{(V_u + \phi_v V_n)}{2}~~ \qquad (6.13.6.1.4b\text{-}2)$$

$$\underline{V_{uw} = \phi_v V_n} \qquad \underline{(6.13.6.1.4b\text{-}1)}$$

where:

ϕ_v = resistance factor for shear specified in Article 6.5.4.2

~~V_u = shear due to the factored loading at the point of splice (kip)~~

V_n = nominal shear resistance determined as specified in Articles 6.10.9.2 and 6.10.9.3 for unstiffened and stiffened webs, respectively (kip)

C6.13.6.1.4b

Delete the 1st Paragraph

~~Eqs. 6.13.6.1.4b-1 and 6.13.6.1.4b-2 provide a more consistent design shear to be used for designing web splice plates and their connections at the strength limit state than that given in past editions of the Standard Specifications and the First Edition of the LRFD Specifications. Eq. 6.13.6.1.4b-1 arbitrarily limits the increase in the shear at the point of splice to 50 percent of the shear due to the factored loading, V_u, where V_u is less than 50 percent of the factored shear resistance, $V_r = \phi_v V_n$, at the point of splice. The increase in the shear is limited to 50 percent of V_u because the possibilities for V_u to change from its calculated value are less than for moment; large unintended shifts in the shear at the splice are unlikely. In addition, the maximum shear is usually not concurrent with the maximum moment at the splice. Thus, the use of a lower value of the design shear in regions where the applied shear is low is deemed reasonable. A lower value of the design shear is also more reasonable for rolled beams, which have significantly higher values of factored shear resistance. For cases where V_u is greater than 50 percent of V_r, the design shear is determined from Eq. 6.13.6.1.4b-2 as the average of V_u and V_r. For checking slip of the bolted connections, the design shear is simply taken as the shear at the point of splice under Load Combination Service II defined in Table 3.4.1-1. The web with the smallest nominal shear resistance on either side of the splice should be used to determine the design shear.~~

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C6.13.6.1.4b

Revise Eqs. (C6.13.6.1.4.b-1) and (C6.13.6.1.4b-2) as follows:

For compact sections:

$$M_{uw} = \phi_f \frac{t_w F_{yw}}{4} \left(D^2 - 4 y_o^2 \right) \qquad \text{(C6.13.6.1.4b-1a)}$$

$$H_{uw} = \phi_f \left(2 t_w y_o F_{yw} \right) \qquad \text{(C6.13.6.1.4b-2a)}$$

For noncompact sections:

$$M_{uw} = \phi_f \frac{t_w D^2}{12} \left(F_{nc} + F_{yw} \right) \qquad \text{(C6.13.6.1.4b-1b)}$$

$$H_{uw} = \phi_f \frac{t_w D}{2} \left(F_{yw} - F_{nc} \right) \qquad \text{(C6.13.6.1.4b-2b)}$$

$$\cancel{M_{uw} = \frac{t_w D^2}{12} \left| R_h F_{cf} - R_{cf} f_{ncf} \right|} \qquad \text{(C6.13.6.1.4b-1)}$$

$$\cancel{M_{uw} = \frac{t_w D}{2} \left(R_h F_{cf} + R_{cf} f_{ncf} \right)} \qquad \text{(C6.13.6.1.4b-2)}$$

where:

t_w =   web thickness of the smaller section at the point of splice (in.)

D =   web depth of the smaller section at the point of splice (in.)

$\cancel{R_h}$ =   ~~hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which F_{cf} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0~~

$\cancel{F_{cf}}$ =   ~~design stress for the controlling flange at the point of splice specified in Article 6.13.6.1.4c; positive for tension, negative for compression (ksi)~~

$\cancel{R_{cf}}$ =   ~~the absolute value of the ratio of F_{cf} to the maximum flexural stress, f_{cf}, due to the factored loads at the midthickness of the controlling flange at the point of splice, as defined in Article 6.13.6.1.4c~~

$\cancel{f_{ncf}}$ =   ~~flexural stress due to the factored loads at the midthicknes of the noncontrolling flange at the point of splice concurrent with f_{cf}; positive for tension, negative for compression (ksi)~~

$\underline{F_{nc}}$ =   nominal flexural resistance of the compression flange at the point of splice as specified in Article 6.10.8.2 (ksi)

$\underline{F_{yw}}$ = specified minimum yield strength of the web at the point of splice (ksi)

$\underline{y_g}$ = distance from the mid-depth of the web to the plastic neutral axis (in.)

$\underline{\phi_f}$ = resistance factor for flexure specified in Article 6.5.4.2

Revise the 4th Paragraph as follows:

In Eqs. <u>C6.13.6.1.4b-1a to C6.13.6.1.4b-2b,</u> ~~C6.13.6.1.4b-1 and C6.13.6.1.4b-2~~, it is suggested that M_{uw} and H_{uw} be computed by conservatively using the <u>flexural resistance</u> ~~stresses at the midthickness~~ of the compression flanges <u>and specified minimum yield strength of the web</u>. ~~By utilizing the stresses at the midthickness of the flanges, the same stress values can be used for the design of both the web and flange splices, which simplifies the calculations. As an alternate, however, the stresses at the inner fibers of the flanges can be used. In either case, the stresses are to be computed considering the application of the moments due to the appropriate factored loadings to the respective cross sections supporting those loadings. In Eqs. C6.13.6.1.4b-1 and C6.13.6.1.4b-2,~~ ~~the concurrent flexural stress at the midthickness of the noncontrolling flange is factored up in the same proportion as the flexural stress in the controlling flange in order to satisfy the general design requirements of Article 6.13.1. The controlling and noncontrolling flanges are defined in Article C6.13.6.1.4c.~~

Revise the 6th Paragraph as follows:

Eqs. C6.13.6.1.4b-1<u>c</u> and C6.13.6.1.4b-2<u>c</u> can ~~also~~ be used to compute values of M_{uw} and H_{uw} to be used when checking for slip of the web ~~bolts~~ <u>connections</u>. ~~However, the following substitutions must first be made in both equations:~~

- ~~Replace F_{cf} with the maximum flexural stress, f_s, due to Load Combination Service II at the midthickness of the flange under consideration for the smaller section at the point of splice,~~
- ~~Rreplace f_{ncf} with the flexural stress, f_{os}, due to Load Combination Service II at the midthickness of the other flange at the point of splice concurrent with f_s in the flange under consideration, and~~
- ~~Set the factors R_h and R_{cf} equal to 1.0. It is not necessary to determine a controlling and noncontrolling flange when checking for slip. The same sign convention applies to the stresses.~~

$$\underline{M_{uw} = \frac{t_w D^2}{12}\left|f_s - f_{os}\right|}$$ (C6.13.6.1.4b-1c)

$$\underline{H_{uw} = \frac{t_w D}{2}\left(f_s + f_{os}\right)}$$ (C6.13.6.1.4b-2c)

<u>where:</u>

$\underline{f_s}$ ≡ <u>maximum flexural stress due to Load Combination Service II at the extreme fiber of the flange under consideration for the smaller section at the point of splice (positive for tension and negative for compression) (ksi)</u>

$\underline{f_{os}}$ ≡ <u>flexural stress due to Load Combination Service II at the extreme fiber of the other flange of the smaller section at the point of splice with f_s in the flange under consideration (positive for tension and negative for compression) (ksi)</u>

<u>In Eqs. C6.13.6.1.4b-1c and C6.13.6.1.4b-2c, it is suggested that M_{uw} and H_{uw} be computed by conservatively using the stresses at the extreme fiber of the flanges. As an alternate, the stresses at the midthickness of the flanges or the inner fibers of the flanges can be used. In either case, the stresses are to be computed considering the application of the moments due to the appropriate factored loadings to the respective cross-sections supporting those loadings.</u>

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6.13.6.1.4c—Flange Splices

Revise as follows:

At the strength limit state, splice plates and their connections on ~~the controlling~~ flanges shall be proportioned to provide a minimum resistance taken as the design stress, F_{cf}, times the smaller effective flange area, A_e, on either side of the splice, where F_{cf} is defined as:

$$\sout{F_{cf} = \frac{\left(\frac{|f_{cf}|}{R_h} + \alpha\phi_f F_{yf}\right)}{2} \geq 0.75\alpha\phi_f F_{yf}}$$

$$\underline{F_{cf} = \alpha\phi_f F_{yf}} \qquad\qquad (6.13.6.1.4c\text{-}1)$$

in which:

A_e = effective area of the flange (in.^{2}). For compression flanges, A_e shall be taken as the gross area of the flange. For tension flanges, A_e shall be taken as:

$$A_e = \left(\frac{\phi_u F_u}{\phi_y F_{yt}}\right) A_n \leq A_g \qquad\qquad (6.13.6.1.4c\text{-}2)$$

where:

f_{cf} = ~~maximum flexural stress due to the factored loads at the midthickness of the controlling flange at the point of splice (ksi)~~

R_h = ~~hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which F_{cf} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0~~

α = 1.0, except that a lower value equal to (F_n/F_{yf}) may be used for flanges where F_n is less than F_{yf}

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

F_n = nominal flexural resistance of the flange (ksi)

F_{yf} = specified minimum yield strength of the flange (ksi)

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ϕ_u = resistance factor for fracture of tension members as specified in Article 6.5.4.2

ϕ_y = resistance factor for yielding of tension members as specified in Article 6.5.4.2

A_n = net area of the tension flange determined as specified in Article 6.8.3 (in.^{2})

A_g = gross area of the tension flange (in.^{2})

F_u = specified minimum tensile strength of the tension flange determined as specified in Table 6.4.1-1 (ksi)

F_{yt} = specified minimum yield strength of the tension flange (ksi)

Delete the 2nd Paragraph

Delete Eq. (6.13.6.1.4c-3)

~~Splice plates and their connections on the noncontrolling flange at the strength limit state shall be proportioned to provide a minimum resistance taken as the design stress, F_{ncf}, times the smaller effective flange area, A_e, on either side of the splice, where F_{ncf} is defined as:~~

$$F_{cf} = R_{cf} \left| \frac{f_{ncf}}{R_h} \right| \geq 0.75 \alpha \phi_f F_{yf} \qquad (6.13.6.1.4c\text{-}3)$$

~~where:~~

~~R_{cf} = the absolute value of the ratio of F_{cf} to f_{cf} for the controlling flange~~

~~f_{ncf} = flexural stress due to the factored loads at the midthickness of the noncontrolling flange at the point of splice concurrent with f_{cf} (ksi)~~

~~R_h = hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which F_{cf} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken as 1.0~~

C6.13.6.1.4c

Delete the 3rd Paragraph

~~The controlling flange is defined as either the top or bottom flange for the smaller section at the point of splice, whichever flange has the maximum ratio of the elastic flexural stress at its midthickness due to the factored loads for the loading condition under investigation to its factored flexural resistance. The other flange is termed the noncontrolling flange. In areas of stress reversal, the splice must be checked independently for both positive and negative flexure. For composite sections in positive flexure, the controlling flange is typically the bottom flange. For sections in negative flexure, either flange may qualify as the controlling flange.~~

Delete the 5th Paragraph.

~~Eq. 6.13.6.1.4c-3 defines a design stress for the noncontrolling flange at the strength limit state. In Eq. 6.13.6.1.4c-3, the flexural stress at the midthickness of the noncontrolling flange, concurrent with the stress in the controlling flange, is factored up in the same proportion as the flexural stress in the controlling flange in order to satisfy the general design requirements of Article 6.13.1. However, as a minimum, the factored up stress must be equal to or greater than $0.75 \alpha \phi_f F_{yf}$.~~

Delete the 7th Paragraph.

~~Since flanges of hybrid sections are allowed to reach F_{yf}, the applied flexural stress at the midthickness of the flange in Eqs. 6.13.6.1.4c-1, 6.13.6.1.4c-3, and 6.13.6.1.4c-5 is divided by the hybrid factor, R_h, instead of reducing F_{yf} by R_h. In actuality, yielding in the web results in an increase in the applied flange stress. When the flange design stress is less than or equal to the specified minimum yield strength of the web, R_h is taken equal to 1.0 since there is theoretically no yielding in the web. The load shedding factor, R_b, is not included in these equations since the presence of the web splice plates precludes the possibility of local web buckling.~~

6.13.6.1.4c— Flange Splices

Revise f_s definition after Eq. (6.13.6.1.4c-5) as follows:

f_s = maximum flexural stress due to Load Combination Service II at the ~~extreme fiber~~ <u>midthickness</u> of the flange under consideration for the small section at the point of splice (ksi)

C6.13.6.1.4c

Revise the 10th Paragraph as follows:

For box section cited in this Article, including sections in horizontally curved bridges, longitudinal warping stresses due to cross-section distortion can be significant under construction and service conditions and must therefore be considered when checking the connections of bolted flange splices for slip for fatigue. The warping stresses in these cases can typically be ignored in checking the top-flange splices once the flange is continuously braced. The warping stresses can also be ignored when checking splices in both the top and bottom flanges at the strength limit state. For these sections, St. Venant torsional shear must also be considered in the design of box-flange bolted splices at all limit states. St. Venant torsional shears are typically neglected in top flanges of tub sections once the flanges are continuously braced. The bolts for box-flange splices may be designed for the effects of the torsional shear using the traditional elastic vector method that is typically applied in the design of web splices. Depending on the limit state under investigation, the shear on the flange bolt group is assumed caused by either the flange force due to the factored loads, or by the appropriate flange design force, as applicable. The moment on the bolt group is taken as the moment resulting from the eccentricity of the St. Venant torsional shear due to the factored loads, assumed applied at the centerline of the splice. At the strength limit state, the torsional shear due to factored loads <u>should be used.</u> ~~need not be multiplied by the factor, R_{cf}, from Eq. 6.13.6.1.4c-3 when computing the moment for the design of the splice.~~ The box-flange splice plates in these cases should also be designed at the strength limit state for the combined effects of the appropriate flange force and the moment resulting from the eccentricity of the St. Venant torsional shear due to the factored loads.

C6.13.6.1.4c

Revise the 11th Paragraph as follows:

In cases for straight girders where flange lateral bending is deemed significant, and for horizontally curved girders, the effects of the lateral bending must be considered in the design of the bolted splices for discretely braced top flanges of tub sections or discretely braced flanges of I-sections. The traditional elastic vector method may also be used in these cases to account for the effects of flange lateral bending on the design of the splice bolts. The shear on the flange bolt group is assumed caused by the flange force, calculated as described in the preceding paragraph. The flange force is calculated without consideration of the flange lateral bending. The moment on the bolt group is taken as the flange lateral bending moment due to the factored loads. At the strength limit state, the flange lateral bending moment due to the factored loads <u>should be used.</u> ~~need not be multiplied by the factor, R_{ef}, from Eq. 6.13.6.1.4c-3 when computing the moment for the design of the splice.~~ Splice plates subject to flange lateral bending should also be designed at the strength limit state for the combined effects of the appropriate flange force and the flange lateral bending moment due to the factored loads. Lateral flange bending can be ignored in the design of top flange splices once the flange is continuously braced.

6.13.6.2—Welded Splices

Revise the 2nd Paragraph as follows:

Welded splices shall be designed ~~to resist the design moment, shear, or axial force specified in Article 6.13.1.~~ at the strength limit state for not less than 100 percent of the factored resistances of the member or element. Tension and compression members may be spliced by means of full penetration butt welds; splice plates should be avoided.

Revise the 3rd Paragraph as follows:

~~Welded field splices should be arranged to~~ ~~minimize overhead welding.~~ Splices, except for orthotropic decks. shall not be field welded.

6.14.2.8—Gusset Plates

Revise as follows:

6.14.2.8.1—General

Gusset plates, fasteners and welds connecting main members shall be designed at the strength limit state for not less than 100 percent of the factored resistances of the member.

Gusset plates, fasteners and welds connecting other members shall be designed at the strength limit state for not less than the factored force effects of the member.

The nominal resistance of a gusset plate shall be based on the effective width as shown in Figure C6.14.2.8.1-1. The gross and net cross-section area shall be on the effective width.

The provisions of Articles 6.13.2, 6.13.3, 6.13.4 and 6.13.5 shall apply, as applicable.

Gusset or connection plates should be used for connecting main members, except where the members are pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member should be given consideration.

Re-entrant cuts, except curves made for appearance, should be avoided as far as practicable.

~~The maximum stress from combined factored flexural and axial loads shall not exceed $\phi_f F_y$ based on the gross area.~~

~~The maximum shear stress on a section due to the factored loads shall be $\phi_v F_u/\sqrt{3}$ for uniform shear and $\phi_v 0.74 F_u/\sqrt{3}$ for flexural shear computed as the factored shear force divided by the shear area.~~

~~If the length of the unsupported edge of a gusset plate exceeds $2.06(E/F_y)^{1/2}$ times its thickness, the edge shall be stiffened. Stiffened and unstiffened gusset edges shall be investigated as idealized column sections.~~
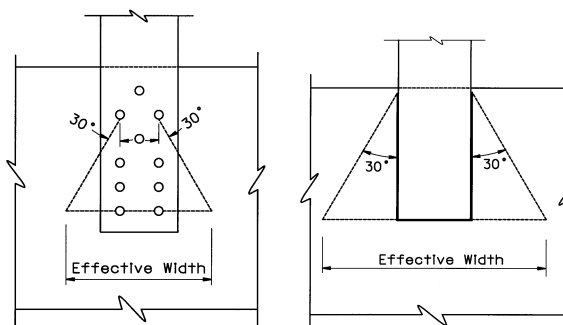
C6.14.2.8

Revise as follows:

C6.14.2.8.1

Major revisions are based on Caltrans successful practice and Caltrans *Guide Specifications for Seismic Design of Steel Bridges* (*Caltrans 2001*).

Figure C6.14.2.8.1-1 shows the effective width for a gusset plate in accordance with Whitmore's method (Whitmore 1952).

(a) Bolted Gusset Plate   (b) Welded Gusset Plate

Figure C6.14.2.8.1-1—Effective Width of Gusset Plate

6.14.2.8.2—Limiting Unsupported Edge Length to Thickness Ratio

C6.14.2.8.2

The unsupported edge length to thickness ratio of a gusset plate shall satisfy:

$$\frac{L_g}{t} \leq 2.06 \sqrt{\frac{E}{F_y}}$$ (6.14.2.8.2-1)

where:

L_g = unsupported edge length of a gusset plate (in.)
t = thickness of a gusset plate (in.)
E = modulus of elasticity of steel (ksi)
F_y = specified minimum yield strength of the gusset plate (ksi)

For stiffened edge, the following requirements shall be satisfied:

- For welded stiffeners, slenderness ratio of the stiffener plus a width of gusset plate equal to ten times its thickness shall be $l/r \leq 40$.
- For bolted stiffeners, slenderness ratio of the stiffener between fasteners shall be $l/r \leq 40$.
- The moment of inertia of the stiffener shall be

$$I_s \geq \begin{cases} 1.83\, t^4 \sqrt{(b/t)^2 - 144} \\ 9.2\, t^4 \end{cases}$$ (6.14.2.8.2-2)

The moment of inertia of the stiffener that is required to develop the post buckling strength of a long plate has been experimentally determined by Eq. (6.14.2.8.2-2) (*AISI 1962*)

where:

I_s = moment of inertia of a stiffener about its strong axis (in.[4])
b = width of a gusset plate perpendicular to the edge (in.)
t = thickness of a gusset plate (in.)

6.14.2.8.3—Tensile Resistance

C6.14.2.8.3

The tensile resistance of a gusset plate shall be:

$$\phi P_n = \phi_y A_g F_y \leq \begin{cases} \phi_u A_n F_u \\ R_r \end{cases}$$ (6.14.2.8.3-1)

This requirement is to ensure that the tensile strength is governed by yielding in the gross section, and fracture in the net section and block shear rupture are prevented.

where:

A_n = net cross-section area of a gusset plate (in.[2])
A_g = gross cross-section area of a gusset plate (in.[2])
        C6.14.2.8.2

F_u = specified minimum tensile strength of the gusset plate (ksi)

ϕ_u = resistance factor for tension fracture in net section = 0.80

ϕ_y = resistance factor for tension yielding in gross section = 0.95

R_r = factored block shear rupture resistance specified by Article 6.13.4

6.14.2.8.4—Compressive Resistance

The nominal compressive resistance of a gusset plate, P_n, shall be calculated in accordance with Article 6.9.4.1.

C6.14.2.8.4

The effective length factor, K in Eqs. (6.9.4.1-1) and (6.9.4.1-2) may be taken as 0.6 for the gusset supported by both edges, and 1.2 for the gusset supported by one edge only (*AISC 2001*); A_s is the average effective cross section area defined by Whitmore's method; l is the perpendicular distance from the Whitmore section to the interior corner of the gusset. For members that are not perpendicular to each other as shown in Figure C6.14.8.2.4-1 (*AISC 2001*), l can be alternatively determined as the average value of

$$l = \frac{L_1 + L_2 + L_3}{3} \qquad \text{(C6.14.2.8.4-1)}$$
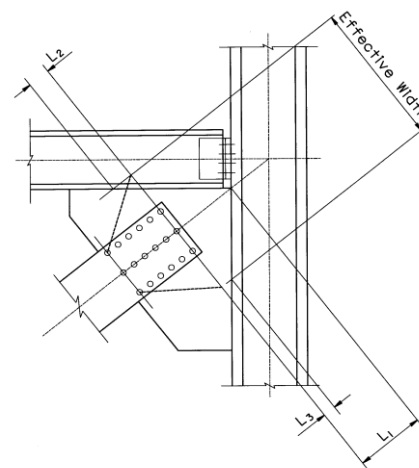

Figure C6.14.2.8.4-1—Gusset Plate Connection

where:

L_1 = distance from the centerline of the Whitmore section to the interior corner of a gusset plate (in.)

L_2, L_3 = distance from the outside corner of the Whitmore section to the edge of a member; negative value shall be used when the part of Whitmore section enters into the member (in.)

6.14.2.8.5—Flexural Resistance

The nominal flexural resistance of a gusset plate, M_n, shall be determined by:

$$M_n = S F_y \qquad\qquad (6.14.2.8.5\text{-}1)$$

where:

S = elastic section modulus of the cross section of a gusset plate (in.^{3})

6.14.2.8.6—Shear Resistance

The nominal shear resistance of a gusset plate, V_n, shall be determined by:

$$V_n = 0.58 F_y A_g \qquad\qquad (6.14.2.8.6\text{-}1)$$

where:

A_g = gross cross-section area of a gusset plate (in.^{2})

6.14.2.8.7—Yielding Resistance under Combined Flexural and Axial Force Effects

The Whitmore's effective area and other critical areas of a gusset plate subjected the combined flexural and axial force effects shall satisfy the following equation:

$$\frac{M_{ux}}{\phi_f S_x F_y} + \frac{M_{uy}}{\phi_f S_y F_y} + \frac{P_u}{\phi F_y A_g} \leq 1 \qquad\qquad (6.14.2.8.7\text{-}1)$$

where:

ϕ_f = resistance factor for flexural

ϕ = resistance factor for axial compression = 0.9, for axial tension yielding = 0.95

M_{ux} = factored moment about x-x axis of the gusset plate (k-in.)

M_{uy} = factored moment about y-y axis of the gusset plate (k-in.)

P_u = factored axial force (kip)

S_x = elastic section modulus about x-x axis of the gusset plate (in.^{3})

S_y = elastic section modulus about y-y axis of the gusset plate (in.^{3})

A_g = gross area of the gusset plate (in.^{2})

F_y = specified minimum yield strength of the gusset plates (ksi)

6.14.2.8.8—Out-of-Plane Forces Consideration

For double gusset plate connections, out-of-plane moment shall be resolved into a couple of tension and compression forces acting on the near and far side plates.

For single gusset plate connections, out-of-plane moment and shear are about the weak axis.

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6.17—REFERENCES

Add the following References:

AISI. 1962. *Light Gage Cold-Formed Steel Design Manual*, American Iron and Steel Institute, Washington, DC.

Caltrans, 2001. *Guide Specifications for Seismic Design of Steel Bridges*, First Edition, California Department of Transportation, Sacramento, CA.

Whitmore, R. E. 1952. "Experimental Investigation of Stresses in Gusset Plates," *Bulletin 16*, University of Tennessee, Knoxville, TN.

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